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PHYSICAL MODEL TESTS ON THE STABILITY OF CEMENTED TAILINGS MINE BACKFILL

FINAL REPORT

by

JOHN D. SMITH ENGINEERING ASSOCIATES LTD.

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John D. Smith Engineering Associates Limited

Consulting Mining and Geotechnical Engineers

110 Lakeshore Blvd., Kingston, Ontario, Canada K7M 6R5 Tel. 613 389-5641

CERTIFICATION

I certify that there are no disclosures to submit under D.S.S. 1036 for contract serial number 1SQ79-00207.

John D. Smith,
President,
John D. Smith Engineering
Associates Limited.

John D. Smith Engineering Associates Limited

Consulting Mining and Geotechnical Engineers

110 Lakeshore Blvd., Kingston, Ontario, Canada K7M 6R5 Tel. 613 389-5641

March 5, 1981

Dr. M. Gyenge,
Rock Mechanics Laboratory,
CANMET,
555 Booth St.,
Ottawa, Ont.,
K1A 0G1.

Dear Mike:

Herewith is included our Final Report on the backfill model testing. The report is divided into two parts. The first part is a complete summary of the testing procedures and the results and conclusions - it is written in the form of a publication. The second part includes additional test details for tests carried out since our interim report of 22 December 1980 - combined with the interim report, this part provides complete details of all tests carried out.

You are aware that Richard Olsen, who took charge of the testing and who solved many of the practical problems that developed during the testing program, is in the process of carrying out a more detailed analysis, beyond the scope of the contract, for his M.Sc. thesis at Queen's University. A copy of this thesis could be made available to you for your personal files after it is completed and examined.

In addition to your acceptance of this final report, we would request your approval to submit Part A of this report for publication in the Canadian Geotechnical Journal. Because of its large national and international circulation and in view of the fact that many geotechnical consultants are involved with mine backfill work, we feel that the Canadian Geotechnical Journal would be the most advantageous forum for communication. Perhaps you could offer suggestions for other forums. Reprints of the publication could also be indexed as an EMR research report and could be circulated by EMR if additional communication of the test results is considered appropriate.

We look forward to receiving your comments with regard to the above.

Respectfully submitted,

John D. Smith, M.Sc.,P.Eng.

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encl.

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PART A

MODEL STUDIES ON CEMENTED TAILINGS MINE BACKFILL

by

Robert J. Mitchell,
Department of Civil Engineering, Queen's University,
Kingston, Ontario, Canada, K7L 4NE

and

Richard S. Olsen and John D. Smith,
John D. Smith Engineering Associates Ltd.,
110 Lakeshore Blvd., Kingston, Ontario, K7M 6R5.

ABSTRACT

Physical model tests on cemented tailings and sand backfills were carried out to determine the strength requirements for stability when vertical fill faces are exposed. The results of this testing support a three-dimensional analytical solution developed in the paper. Comparisons with conventional design approaches indicate that substantial savings in cement usage can be realized, for many blast hole stoping configurations, by use of the three-dimensional design approach.

1. INTRODUCTION

Large scale bulk mining methods have been developed to increase the efficiency of underground metaliferous mining operations. The success of bulk mining methods requires that ground control be maintained in order that ore can be continuously recovered according to a planned sequencing. Continuous placement of backfill can provide the required ground control. Waste tailings sands, hydraulically delivered (poured), generally provide the most economical backfill. Stabilization of this type of backfill is necessary in order to extract adjacent ore pillars and the normal method of stabilization is by the addition of normal Portland cement to the tailings pour. The increasingly adverse relation between cement costs and mineral values, however, has led to decreased ore recovery in some mines (a remnant pillar is left to contain uncemented backfill) and to revisions in mining or backfilling methods in other mines to avoid the use of cemented bulk pours. The work reported in this paper is concerned with designing for more efficient and economical use of cemented tailings backfills.

2. THEORETICAL CONSIDERATIONS

Cement stabilized sand fills are used to provide temporary ground support while adjacent ore pillars are removed. As long as free spans are designed to avoid caving, the largest shear stresses in the backfill are due to self-weight and the cemented fill is, therefore, designed as a vertical slope. Two design approaches are in common use: a free standing wall approach where the unconfined compressive strength required at any depth in the fill is given as $[\sigma_1]_F \geq \gamma z$; a vertical slope approach where a constant

strength fill is required to have a cement bond shear strength of $c_b \geq \gamma H/4$ (since the maximum shear stress in an unconfined sample is given, at failure, as $c_b = [\sigma_1]_F/2$, this design requires that $[\sigma_1]_F \geq \gamma H/2$). Unconfined test results are normally used to determine the cement requirements and, since the unconfined strength of a given backfill sand varies approximately linearly with cement content over the practical range of 3% to 8% cement by dry weight, both approaches give approximately the same overall cement usage. The free-standing wall approach, however, results in a variable strength fill (strength increases linearly with depth) with an increased risk of ore dilution due to surficial failures resulting from blasting, closure or surcharge.

Neither of the above design approaches account for the true three-dimensional geometry of the cemented fill block. As shown on Figure 1, a portion of the block weight may be resisted by shear along the rough rock wall contacts. A simplified evaluation of this wall effect for drained, cemented backfill material can be carried out by assuming that a constant wall shear stress (equal to the cement bond shear strength) is mobilized to reduce the net effective weight of the sliding block as follows:

Net weight of block = $W_n = wH^*(\gamma L - 2c_b) + q$

Factor of safety = $\frac{\text{resisting forces in failure plane}}{\text{driving forces on failure plane}} = F$, and

for $q = 0$, $$F = \frac{W_n \cos\alpha \tan\phi + cLw/\cos\alpha}{W_n \sin\alpha} \qquad - (1)$$

$$F = \frac{\tan\phi}{\tan\alpha} + \frac{2cLw}{wH^*(\gamma L - 2c_b)\sin 2\alpha}$$

where $H^* = H - \frac{w\tan\alpha}{2}$

c = apparent cohesion on the failure plane

and α is taken to be $45° + \phi/2$ (critical plane).

If it is assumed that $c_b = c$, the required cohesion for any given ϕ angle is given for $H >> W$, as

$$2c = \gamma H \Big/ \left(\frac{H}{L} + 1 \Big/ \left(1 - \frac{\tan\phi}{\tan\alpha}\right) \sin 2\alpha\right) \qquad - (2)$$

The above analysis can be related to the common approaches outlined earlier by noting that the unconfined compressive strength, $[\sigma_1]_F = 2c_b$, required for $\phi = 0$ is given as $[\sigma_1]_F = \frac{\gamma H^*}{(1 + H^*/L)}$. Because the fill height (H) is generally much larger than the fill width (w) and to provide a conservative estimate of the unconfined compressive strength requirement, this equation can be reduced to

$$[\sigma_1]_F = \gamma H \Big/ (1 + H/L) \qquad - (3)$$

It may be noted that equation 3 gives the same answer as the vertical slope approach when H = L but requires a much reduced strength when H>>L. When $L \to 0$, no cement is required since the uncemented sand would be self-supporting due to arching. Failure mechanisms more complex than that shown on Figure 1 have also been analyzed (including wedge failures and conical failures) but these analyses have indicated that the confined block mechanism is the most probable failure mechanism.

In the development of equation 1, it is conveniently assumed that no contribution to wall shear can be derived from friction. In this case, the contribution of frictional shearing resistance to stability is small ($\tan\phi/\tan\alpha$ is, for example, equal to 0.33 for a typical ϕ of 30°). Equation 3 assumes that the cemented sand backfill is a frictionless material ($\phi = 0$). It is of interest to examine the applicability of this simplifying assumption from both theoretical considerations or laboratory test results.

It is well known from test results that cement bond strength is developed, in a fully cured cemented sand at very low strains while much larger strains are required to develop frictional shearing resistance. Furthermore, it is known that the cement bonds will not crush under confined compressive loading unless the major principal stress exceeds the unconfined compressive strength; test data published by Mitchell and Smith (1979) indicates that the confined one-dimensional (oedometer) crushing strength is at least 2.5 times the unconfined strength. The major principal stress in a poured fill is limited by arching when $H > L$, to a maximum value of $\sigma_1 = \gamma L/2\tan\phi$ (Terzaghi, 1943; Barrett et al, 1978, Askew et al, 1980). Since friction will be mobilized prior to curing, the maximum stress must generally be less than $\sigma_1 = \gamma L$. Therefore, to avoid cement bond crushing in a fill designed using equation 3,

$$\gamma L \leq \frac{2.5\ \gamma H}{1 + H/L}$$

giving $L \leq 1.5H$. This calculation indicates that for all cases where equation 3 would be used in design (i.e. where $H > L$), crushing of cement bonds would not occur and, therefore, cement bond resistance would be mobilized, in shear, at small strains. These theoretical considerations support the use of a $\phi = 0$ design for cemented fill.

Figure 2 shows the results of shear box tests carried out on cemented samples of a tailings sand having an effective grain size, D_{10}, of 0.026 mm and a uniformity coefficient of 3.2. The data indicates that a constant cement bond strength can be mobilized under small normal stresses. Under larger normal stresses the cement bond strength is still mobilized at small strains but, at large strains, frictional resistance is developed. Unconfined

compressive strength results, also plotted on Figure 2, confirm the cement bond strength values. Data points on Figure 2 could, however, be approximated by two strength parameters, c and ϕ. In this case, an analysis of the failure mechanism of Figure 1, assuming that water pressures do not exist, would give the net weight of the block (without surcharge) as

$$W_N = \gamma LwH^* \left[1 - \frac{2c}{\gamma L} - \frac{1}{2}\left(1 - \frac{L}{H}\left(1 - 2 K_c \tan\phi\right)\right)\right]$$

Approximating $K_c = 1-\sin\phi$, the value of $(1 - 2 K_c \tan\phi)$ is found to be 0.4 for $30° \leq \phi \leq 40°$ and the frictional wall resistance would vary from 0.3 γLwH^* when L = H to 0.5 γLwH^* when $L \to 0$. Furthermore, it can be shown from the geometry of the Mohr stress circle that $c = \frac{C_b(1 - \sin\phi)}{\cos\phi}$ which gives values of c equal to 0.577 C_b, 0.52 C_b and 0.466 C_b for ϕ = 30°, 35° and 40°, respectively. Then, for a typical backfill with ϕ = 35° and H = 3L, the net block weight is given by $\gamma LwH^* \left(1 - \frac{C_b}{L} - 0.43\right)$. This can be compared with $\gamma LwH^* \left(1 - \frac{2C_b}{\gamma L}\right)$, for a ϕ = 0 backfill, by making use of equation 3 (which gives $2C_b = [\sigma_1]_F = {}^{\gamma H}/4$ for the condition H = 3L). For this typical example, the above equations predict that the net block weight would be decreased by 20% if both c and ϕ were mobilized in wall shear (as compared to the case where only C_b was mobilized).

From the above considerations, it is concluded that the constant wall shear assumption, as used in deriving equations 1, 2 and 3, is a satisfactory assumption (slightly conservative). Equation 1, then, should give a good estimate of the factor of safety of a fill but requires both confined and unconfined test results. Equation 2 will yield a theoretically conservative design from the results of confined triaxial or shear box tests.

Equation 3, which requires only unconfined test results, is considered to be the most realistic equation for the design of cemented tailings backfills and is used to correlate with the physical model studies presented in this paper.

The preceding theoretical developments do not consider the possible effects of closure strains which might occur when the adjacent ore pillar is extracted. Because the stiffness of the backfill is small (10^4 to 10^5 kPa) compared to that of rock (10^7 to 10^8 kPa), the strains imposed on the fill will not, for normal fill strike lengths, be sufficient to cause cement bond rupture in the fill (i.e. $^{\Delta L}/L$ will generally be less than 1%). Visual evidence from unconfined compression tests indicates that tensile cracking and associated surface spalling develops only when the maximum unconfined shear strength is approached (i.e. both shear failures and tensile cracking can develop at about 1% axial strain). Thus, in normal situations, no fill failures would be expected due to closure strains. Since fill strength increases with confining (normal) stresses, closure might be expected to provide a marginal increase in fill stability. At worst, closure strains may result in failure developing on planes of lower normal stress producing wedge or conical failures with no change in fill stability. Several model tests reported herein were carried out under 1% closure strain in order to evaluate these hypotheses.

3. PHYSICAL MODEL STUDIES

3.1 Dimensionless factors and model scale

The dimensionless factors $\frac{C_b}{\gamma H}$, tanϕ, $\frac{H}{L}$ and $\frac{H}{w}$ have been determined theoretically to govern the stability of an exposed vertical face of cemented backfill. Even if the frictional factor (tanϕ') is considered to be of secondary importance due to strong cement bonding, there are obvious advantages to using identical materials in model and prototype. The selected materials, a clean washed sand and a fine silty sand tailing product, are considered to be representative of prototype tailings sand backfills as noted on Figure 3. Linear geometrical factors can be adequately modelled and it remains to satisfy similitude of the factor $\frac{c}{\gamma H}$. Prototype backfills are sufficiently high that strength effects due to capillary moisture can be ignored, they have a relatively high cement bond strength such that frictional strength is thought to play a secondary role in stability and they are usually sufficiently well drained that pore water pressures are equal to zero. Unless centrifugal forces are used to increase γ (Schofield, 1978), reduction of the height of exposure, H, to a model scale requires proportional reduction of the cement bond cohesion, c_b. At model heights, the apparent strength due to capillarity in these materials would have a significant effect. This effect is eliminated by maintaining 100% saturation in the model. Pore water pressures are then reduced to zero everywhere in the homogeneous saturated model by opening base drainage facilities to create a downward hydraulic gradient of unity just prior to exposing the vertical face. This creates an effective unit weight γ equal to the saturated unit weight of the material. The cement bond cohesion can be effectively

reduced by using a nominal cement content (2.2% was found to be sufficient to produce a uniform material) and limiting the curing time. Figure 4 shows laboratory test data relating strength of cemented samples to curing times up to 24 hours. From equation 3, it is estimated that a prototype fill of 40 m. height would require a maximum shear strength, at L = H, of about 200 kPa (an unconfined compressive strength of 400 kPa). If one day of curing was allowed in the sand model, for example, the data on Figure 4 indicates that the 24 hour shear strength would be about 25 kPa, producing a scale factor of 8. The resulting model would then be $^{40}/_{8}$ = 5 metres in height. Although it is advantageous to do model tests at as large a scale as practicable, a test box of 5 metres height was not within the scope of this study. To satisfy space and time restrictions, a nominal model height of 2 metres was selected, resulting in a scaling factor, in comparison to typical prototype backfills, of the order of 20 to 50. At this reduced scale, fill faces required exposure after only a few hours of curing in order to create model fill failures. A reduced scale, of course, reduces the material volumes required and it was possible to mix and pour individual models within a time frame of 20 to 40 minutes using an ordinary mortar mixer. Even at this scale, approximately 12 tonnes of sand materials was used in completing 30 model tests.

3.2 Model test facility

In order to vary linear lateral dimensions and allow closure strains to be applied while exposing a test fill, a braced steel frame (2 m. high by 1.8 m. wide by 0.9 m. deep) was constructed to support the model formwork. Two-inch thick laminated plywood was used to form a fixed back and

a fixed bottom in the frame. General views of the model frame, without and with interior formwork, together with the mixing facilities, are shown on Figure 5. To adjust the strike length, L, of the model, two-inch thick laminated plywood sides, which were fabricated in the required widths (0.2 m., 0.4 m., and 0.6 m.), were supported by rectangular steel tubing bearing on timbers supported by the steel frame. This detail is shown on Figure 6. The adjustment bolts are used to apply closure strains ($^{\Delta L}/L$) to the model fills. The front of the box was made up of individual, 10 cm square section, horizontal timbers supported by tightening bolts from steel box sections bearing on the steel frame as can be seen on Figure 5. Sealing of the box was accomplished by a combination of waterproof caulking, rubber straps and plastic sheeting, as detailed on Figure 7. Figure 7 also shows details of the base drainage. Rock wall roughness was modelled by tacking triangular wood pieces to the side walls of the formwork (protruding about 2 cm) as seen on Figure 5a. Two coats of epoxy paint were applied to all plywood sections and steel supports were painted to prevent oxidation. The rubber straps (Figure 7), as well as affecting a watertight seal when the front timbers were tightened into place, deformed easily to allow closure strains to be applied by lateral movement of the side formwork.

3.3 Model test procedures and problems

Following the erection of formwork for a particular test, the model fill is poured. Pre-weighing of materials and efficient job organization resulted in average pour durations of 30 minutes for a three-man team. Mixing and pouring was done at a pulp density (P.D. = mass of dry solids/ mass of slurry) of about 70% leaving little excess water on the top of the

fill after pouring. When the excess water drained through, the drainage was closed to preserve the fill saturation. Control samples were poured into short sampling tubes during each pour and were tested at predetermined times to determine when the correct fill strength had been attained. Two pieces of testing apparatuses (shown on Figures 8 and 9 were developed to evaluate the short term strength of control samples and of samples taken after the fill was exposed. Formed samples or samples taken from cast fills are extruded into either test apparatus and subjected to unconfined (Figure 8) or direct shear (Figure 9). These tests were done rapidly, reaching failure in about one minute. (The measurement accuracy is $\pm$0.1 kPa in shear strength). Density and moisture content determinations were also carried out on all samples taken from cast fills. When the required strength was achieved, after 2.5 to 5 hours of curing, the drainage valve was opened and the front timbers were removed, exposing the vertical fill face. Normally, timbers were removed from top to bottom at a rate such that the entire height was exposed in less than 10 minutes. Removal of timbers from bottom to top was done in two tests and the timbers were oriented in the vertical plane to allow side to side exposure in two tests. While the height of pours was kept constant (settled height of approximately 1.8 m) the formwork was reconstructed as required to provide various ratios of $\frac{H}{L}$ and $^{H}/_{w}$. The aim of the testing procedures was to produce fill failures that encompassed the full height of the exposed fill. In most tests a failure condition was achieved, during removal of the front timbers, at heights less than 1.8 m. For analysis purposes, the overall height of each failure was used in calculating the values of the dimensionless factors at failure.

Three main problems were encountered in the model test program. Temperature variations in the building altered curing rates making control difficult. This problem was solved by providing a plastic tent and electrical heater to maintain nearly constant temperature during curing. Mixing water was also heated prior to a pour. The second problem developed only with the coarser, more pervious sand and involved seepage erosion (sapping) of the exposed face during the removal of the front timbers and plastic sheet. This problem was solved by using double stick carpet tape to stick the clear plastic sheet to the front edge of the side walls. This prevented air from entering the exposed face and thus, water bleeding from the exposed face was minimized. By casting a thin layer of slimes on the pour surface (in addition to the plastic sheet), the seepage erosion was eliminated. When fill failure was imminent, as indicated by bulging of the exposed face and cracking within the cast fill, the plastic was quickly pulled free to allow the failure to develop unrestricted. The third problem was more difficult, namely that direct shear and unconfined compression tests on very weak mixes gave different values of shear strength. Both types of testing were continued throughout the model testing program and it was noted that the direct shear results were quite consistent while the compression results were subject to considerable scatter. A detailed laboratory program carried out during the latter part of the model testing led to a clarification of the strength problem. It was found that the direct shear strength during early curing was increased by increases in total normal stress. These test results are plotted on Figure 10. In addition, careful testing revealed that unconfined test samples exhibited a yield stress followed by increased

resistance to load (work hardening). This work hardening is explained by two factors: increased shear area due to bulging and increased normal stress on potential failure planes as the load is increased. Figure 11 shows, for a number of unconfined tests, the estimated yield stress and ultimate stress (load over corrected area) together with the corresponding shear and normal stresses on the planes of failure (samples failed at angles between 50° and 65° to the major principal plane). These yield stress values of shear and normal stresses are shown on, Figure 10, to confirm the direct shear test data. Based on these correlations, the results of direct shear tests on control samples and samples taken from cast models are used in analyzing the model test results.

3.4 Test program and test results

At the beginning of the test program, several tests were allowed to cure to excessive strengths and did not fail. The sequence of photos on Figure 12 shows the result of test T11, one of the tests that exhibited a full height failure. The first visual sign of distress in most tests is shear cracks forming along the side walls. As the mass slumps, there is noticeable bulging associated with the development of a failure plane. Finally, the mass accelerates out of the form and comes to rest on the floor. The final picture on Figure 12 shows the failure surface after the failed mass was removed. This failure surface is slightly concave with an average angle close to 65°. Note, from Figure 12, that edge shear leaves a layer of cemented fill in contact with the rough walls but that the fill separates cleanly from the back wall (in this particular test, a clear plastic sheet

was placed along the back wall prior to pouring but clean separation was also noted in the absence of this plastic sheet). Indeed, the failure pictured on Figure 12 corresponds very well with the mechanism of failure assumed on Figure 1.

Table 1 lists some relevant data from all of the tests carried out. Tests T2A and S3 were poured without cement and allowed to settle for several hours to ensure that soil arching would not prevent failure. A continuous slumping failure, as each timber was removed, was observed in these tests. In several tests, when full exposure did not result in a failure, additional load (sand bags) were placed immediately on the top of the fill to create a surcharge. Test T6, for example, required a surcharge of about 45 kg to cause failure. The resulting failure, in which the failed mass actually toppled out of the form after lateral shear failure planes had developed, is shown on Figure 13. In several of the tests, when an initial failure occurred in the upper half of the fill, (i.e. when the exposed height was less than one metre), a second failure of similar height was created upon removal of the remaining facing timbers. Test S17, pictured on Figure 14, is an example of this behaviour. In test S17, the first failure resembles, to some degree, a circular arc failure as might be expected when the strike length (L) exceeds the failure height (H) and the width of the fill (w) is sufficient for a circular arc failure to be kinematically possible. Greater curvature in the failure surface also appeared to be promoted by the application of closure strains. Test T10, Figure 15, had 1% closure strain [$(^{\Delta L}/L) = 0.01$] applied just prior to exposure.

Test T11, which was exposed from the bottom up, produced a very

similar failure (see Figure 12) to that observed in an identical fill exposed in the normal manner (test T9). Figure 16 shows the failure in test T9. Wall roughness members were removed in test S18 to see what effect this roughness had on failure and, as shown on Figure 17, the failed mass separated cleanly from the side walls. Tests S27 and S28 were exposed from one side to the other and produced, as shown on Figure 18, a failure which was similar to other observed failures.

3.5 Analysis of test results

The average of the direct shear strength measurements done on samples taken from either the intact parts of a failed block or from the material left in the formwork after each test was completed (normally 4 to 5 tests) are tabulated in Table 1. Also listed, are the average direct shear strengths obtained from control tests (usually two tests) carried out just before fill exposure. All of these tests were carried out under a normal stress of 5kPa. By estimating the average normal stress on the observed failure surface for each observed failure, the most probable average shear strength mobilized during failure has been estimated using the slope of the relations plotted on Figure 10. These most probable strengths, listed as corrected strengths in Table 1, are also considered to be the highest estimate of the cement bond strength mobilized in the test fills and were used to calculate the values of $2c_b/\gamma H_f$ listed in Table 1. Figure 19 shows the correlation between equation 3 and the model test results. This correlation indicates that equation 3 adequately describes the stability of the model test cemented fills. Test T26 is the only data point that lies significantly above the theoretical curve and five cast fills which did not fail

add confidence to the correlation (in particular, tests S19 and S21). Within the limitations of the test program, it is observed that neither closure strain (1% closure used) nor method of exposure had any significant effect on the stability. Equation 1 does indicate that the fill width, w, will have a significant effect on the stability in cases where w is the same order of magnitude as the fill height (or failure height) and $^{H}/_{L}$ is less than about 2.0. As $^{H}/_{w}$ decreases, the strength requirement is decreased. This prediction is supported by the test data: tests S7, S8 and T9 have $^{H}/_{w}$ less than 3.5 while tests S13 through S18 have $^{H}/_{w}$ values between 4 and 5. In practice, for $H \leq 2L$, less cement would be indicated when the fill is backed by ore than when the fill is backed by previously poured and drained fill. Figure 20 shows the failure of a model backfill with H/L greater than 4 and H/w greater than 8.

4. ECONOMIC AND OPERATIONAL CONSIDERATIONS

It is not uncommon for fill heights to exceed the lateral mining dimension (L) in underground metaliferous mines using blast hole stoping methods and, indeed, extraction with H>>L could be planned, if proven economical. Consider an opening mined to H = 4L as opposed to 4 stoping blocks with H=L: equation 3 predicts that the cement strength requirement would increase by only 60% by having H=4L (instead of H=L) while the common approaches would predict a 400% increase in cement strength requirement. A 60% increase in cement costs could be reclaimed many times over by the 60% reduction in sub-level development work. Furthermore, if equation 3 is accepted, an existing mine with H=4L could be spending twice as much money on cement as required. At a backfill pouring rate of 4,000 tonnes per day,

and assuming 50% of this fill is cemented at 5% cement by dry weight (20:1 T:C ratio), the annual cement usage would be over 35,000 tonnes. At 35 dollars a tonne delivered, a savings of over half a million dollars per year could be realized if a 40:1 tailings to cement ratio could be shown to be stable. Of even greater importance to the overall economics of the mining industry is the strong probability that economical use of cemented mine tailings backfills could improve the efficiency and recovery in underground mining operations. Tailings used underground as fill reduces the volumes of wastes to be disposed of on surface, with an attendant reduction in environmental problems.

In applying equation 3 (or any other stability equation) to prototype design, it must be realized that the uniformity achieved in cast laboratory samples (or cast model tests) is generally not achieved in large bulk pours. The effect of segregation on the mass strength of a prototype backfill is difficult to ascertain although, theoretically, the bulk or mass strength should be the same in segregated or uniform fills providing that the strength is linearly related to the cement content and that no weak slimes layers are created. Weak layers dipping toward the exposed face would, of course, be detrimental to stability. With high pulp densities, good drainage facilities and judicial selection of pour points, adverse segregation can be minimized. The authors are aware of two separate mines in which high stable backfills have been designed using equation 3 and the results of laboratory unconfined compression tests. These fills have calculated safety factors, by this design approach, of less than 1.2 and have performed satisfactorily.

In evaluating the stability of cemented fills, the effects of adjacent blasting should be considered. There is no danger that normal pillar blasting operations will induce mass failure since any reasonable level of cementation strength will eliminate the risk of vibration-induced flow. Good drilling and blasting control can reduce the extent of local fill failures; both ring blasting and cratering techniques have been used successfully in operations employing cemented fill. Cement bonding and rough wall interlocking between the pillar ore and the cured backfill, however, means that either some ore dilution by cemented fill must be accepted or that a very thin skin of ore must be left adhering to the fill. There does not appear to be any evidence to indicate that the cement content of the fill (or the fill strength) has any effect on the quality of ore separation. In the opinion of the writers, the reduced fill stiffness associated with reduced cement content would be beneficial in resisting local blast damage and the reduced ore-fill adhesion would promote separation of the ore and backfill. Certainly, higher than necessary fill cement contents would not be recommended due to any blast damage or ore separation considerations.

The work described in this paper is based on, and advocates the use of, a constant strength cemented backfill. Because strength increases very rapidly with added cement content above about 8% (Weaver and Luka, 1970), greater economies in cemented backfill practice might be achieved by controlled pouring of alternate thick layers of low cement fill and thin layers of high cement fill to produce a reinforced cemented backfill. A detailed analysis of such reinforced systems is, however, outside the scope of this paper.

5. CONCLUSIONS AND RECOMMENDATIONS

A three-dimensional analysis of the stability of cemented tailings backfill is presented and a simplified equation for design purposes is derived as

$$[\sigma_1]_F = {}^{\gamma H}/_{(1 + {}^{H}/_{L})}$$

where $[\sigma_1]_F$ is the unconfined compressive strength of the cemented fill,

γ is the total unit weight of the fill material,

H is the exposed fill height (vertical face),

and L is the strike length of the exposed face.

The results of twenty-six small scale model tests support the three-dimensional analysis. From the success of the small scale model studies and considering the economic advantages to be derived from reduced cement usage, it is concluded that larger scale model pours (5 m. in height) could be justified in the design stage at individual mines.

Although the above simplified equation uses only results from unconfined compression tests, c and ø parameters derived from direct shear or confined triaxial tests can be used in equations developed in the paper (equations 1 and 2) to confirm the safety of the design.

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Test No. S = sand T = tails	L m	w m	H_f m	γ kN/m^3	shear angle	Ave.direct shear strengths, C_b, kPa			$\frac{2C_b}{\gamma H_f}$	$\frac{H_f}{L}$	Comments
						samples	control	corrected			
S1	0.4	0.2	None	19.8	-	6.5	NA	6.5	.36	4.5	
S1A			1.4	20.0	61°	3.2	3.1	3.2	.23	3.5	
T2			None	19.4	-	5.1	NA	5.1	.29	4.5	
T2A			Flow	18.8	-	0	NA	0	-	-	No cement
S3			Flow	19.7	-	0	NA	0	-	-	No cement
S4			0.6	19.7	60°	2.2	2.0	2.2	.37	1.5	
T5	↓	↓	None	19.5	-	5.6	NA	5.6	.31	4.5	
T6	0.8	0.4	1.8	19.5	62°	4.4	NA	4.9	.28	2.25	45 kg surcharge
S7			0.9	19.7	60°	2.7	2.7	2.7	.31	1.13	
S8			1.3	18.8	60°	3.8	3.5	4.0	.31	1.63	
T9			1.4	19.0	72°	3.0	3.5	3.5	.27	1.75	
T10			0.9	18.7	63°	2.8	3.0	3.0	.35	1.13	1% closure
T11			1.6	18.9	65°	4.2	5.0	4.4	.29	2.0	exposed bottom up
T12		0.2	1.8	18.9	65°	4.0	NA	4.5	.26	2.25	45 kg surcharge
S13			0.8	19.5	68°	3.1	3.0	3.3	.43	1.0	two similar failures
S14			1.0	19.3	66°	4.0	4.0	4.0	.41	1.25	
S15	↓		0.8	19.4	69°	2.9	3.4	3.4	.44	1.0	1% closure strain
S16	0.6		0.8	19.3	70°	2.7	3.0	3.0	.35	1.42	two failures
S17			0.9	19.6	56°	2.5	3.25	3.0	.34	1.5	" "
S18		↓	0.8	19.4	66°	3.2	3.2	3.2	.41	1.33	" "
S19		0.4	None	20.0	-	4.1	3.75	4.1	.23	4.5	>300 kg surcharge
S20			1.8	18.5	60°	3.4	3.5	3.6	.22	3	
S21	↓		None	18.0	60°	3.9	4.0	4.4	.27	3	1% closure strain
T25	0.4		1.4	18.0	72°	2.9	NA	2.9	.22	3.5	
T26	0.4	↓	1.7	18.2	66°	3.1	NA	3.2	.21	4.25	exposed bottom up
S27	0.6	0.2	1.5	18.3	78°	3.5	3.5	3.6	.26	2.5	lateral exposure
S28	0.6	0.2	1.5	18.1	66°	3.6	3.75	3.85	.28	2.5	lateral exposure

TABLE 1 - SUMMARY OF MODEL TEST RESULTS

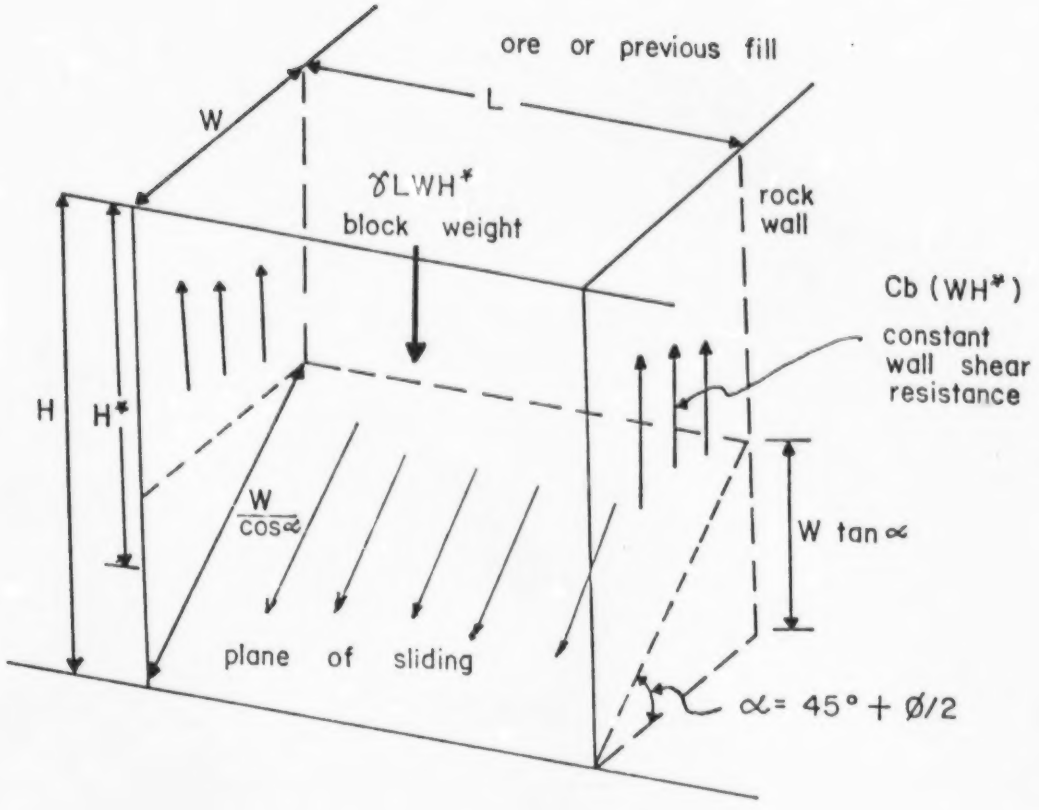


FIGURE 1 CONFINED BLOCK MECHANISM

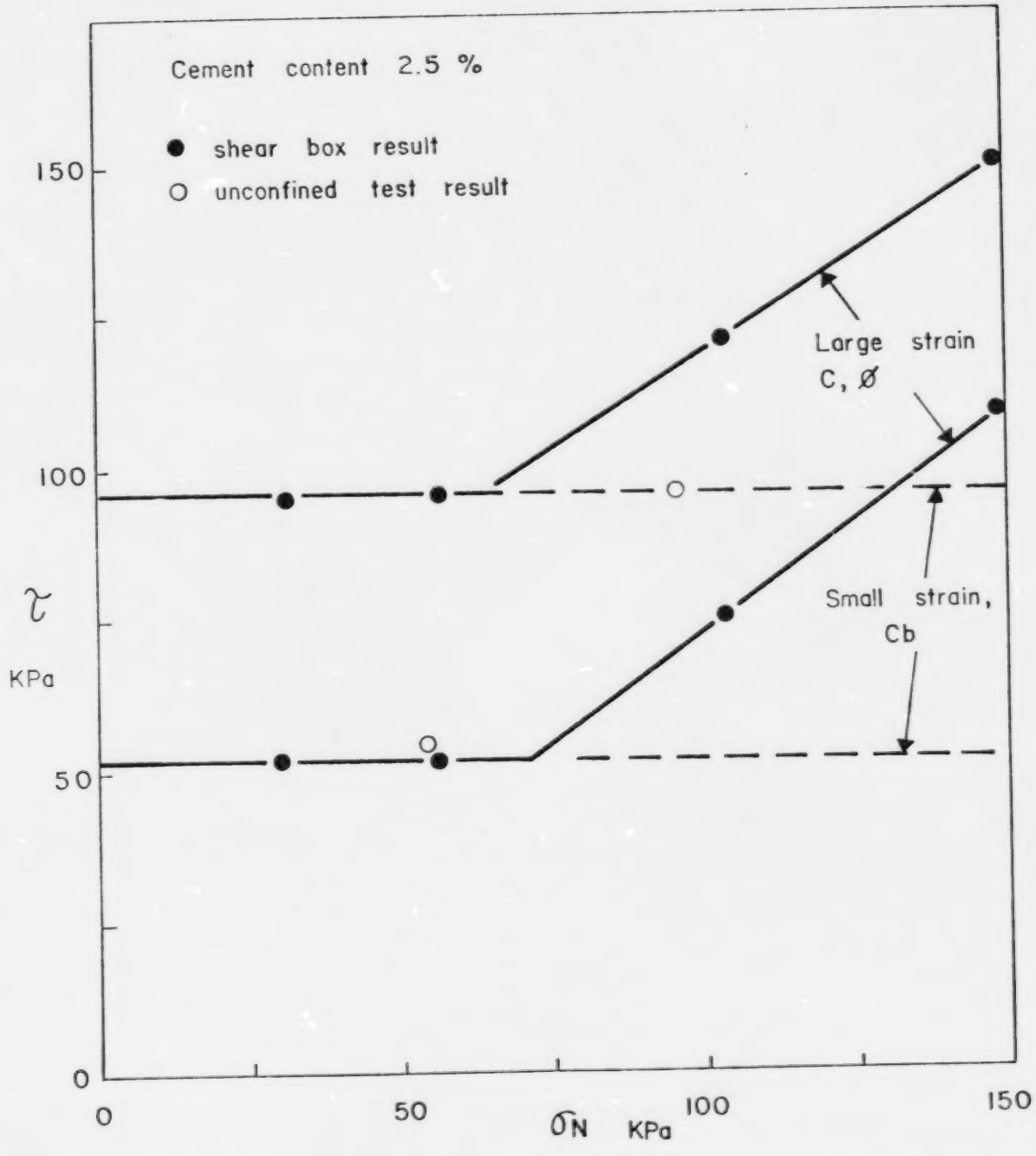


FIGURE . 2 SHEAR BOX TEST RESULTS

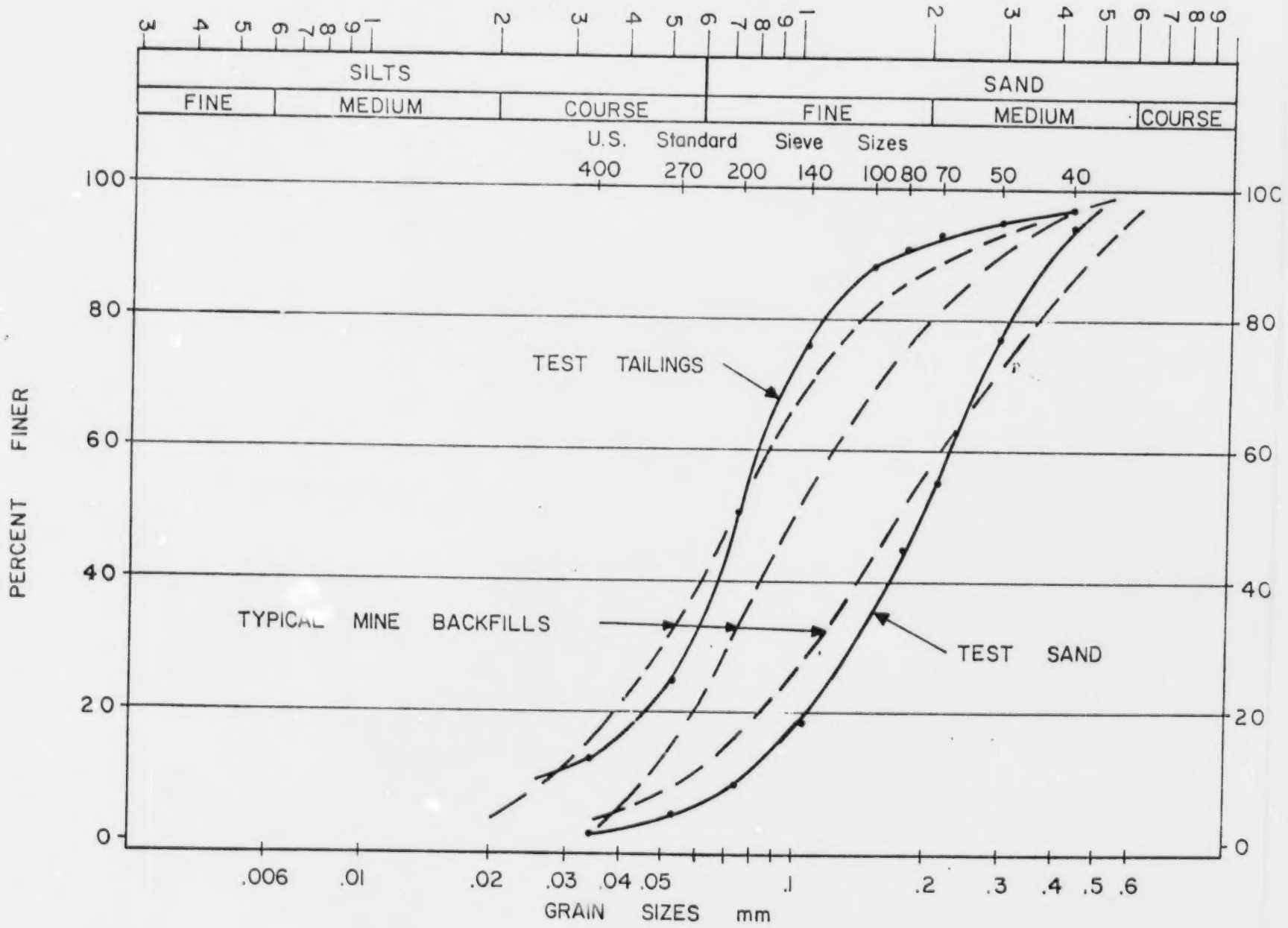


FIGURE 3 GRAIN SIZE DISTRIBUTIONS OF SANDS AND TAILS

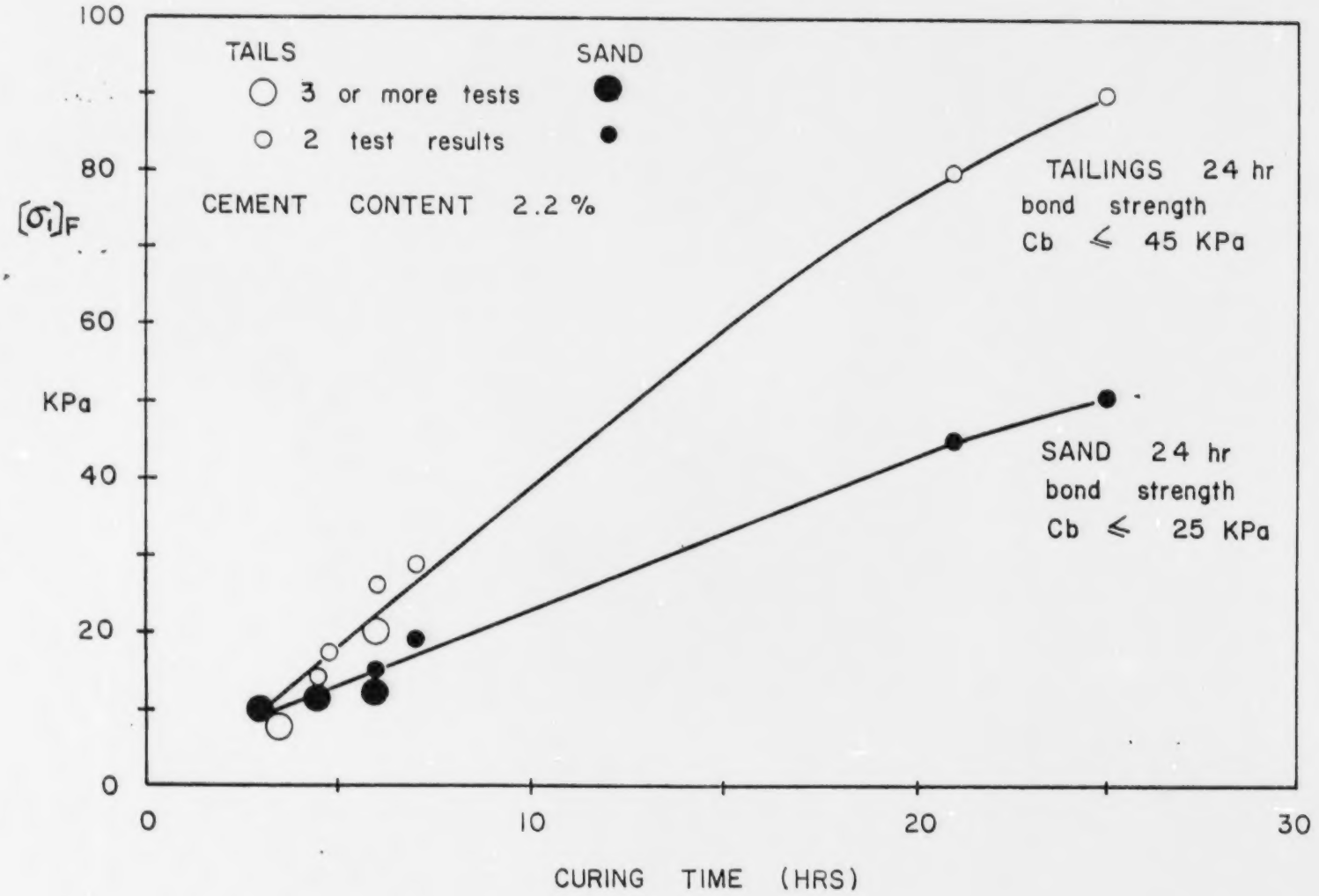
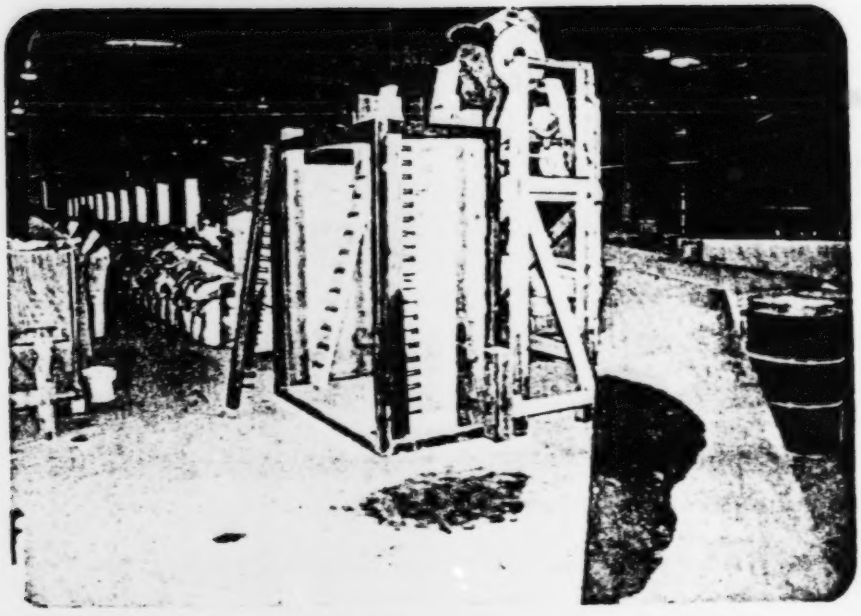
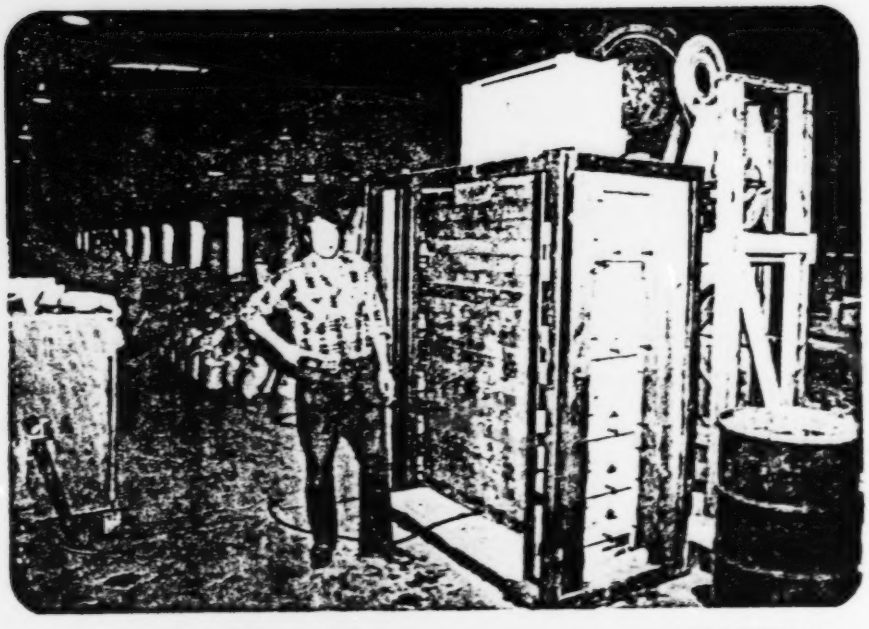


FIGURE 4 UNCONFINED COMPRESSION TEST RESULTS



a) TEST FRAME



b) MODEL FILL CAST IN FORMWORK

FIGURE 5 — GENERAL VIEWS OF MODEL FRAME AND FORMWORK

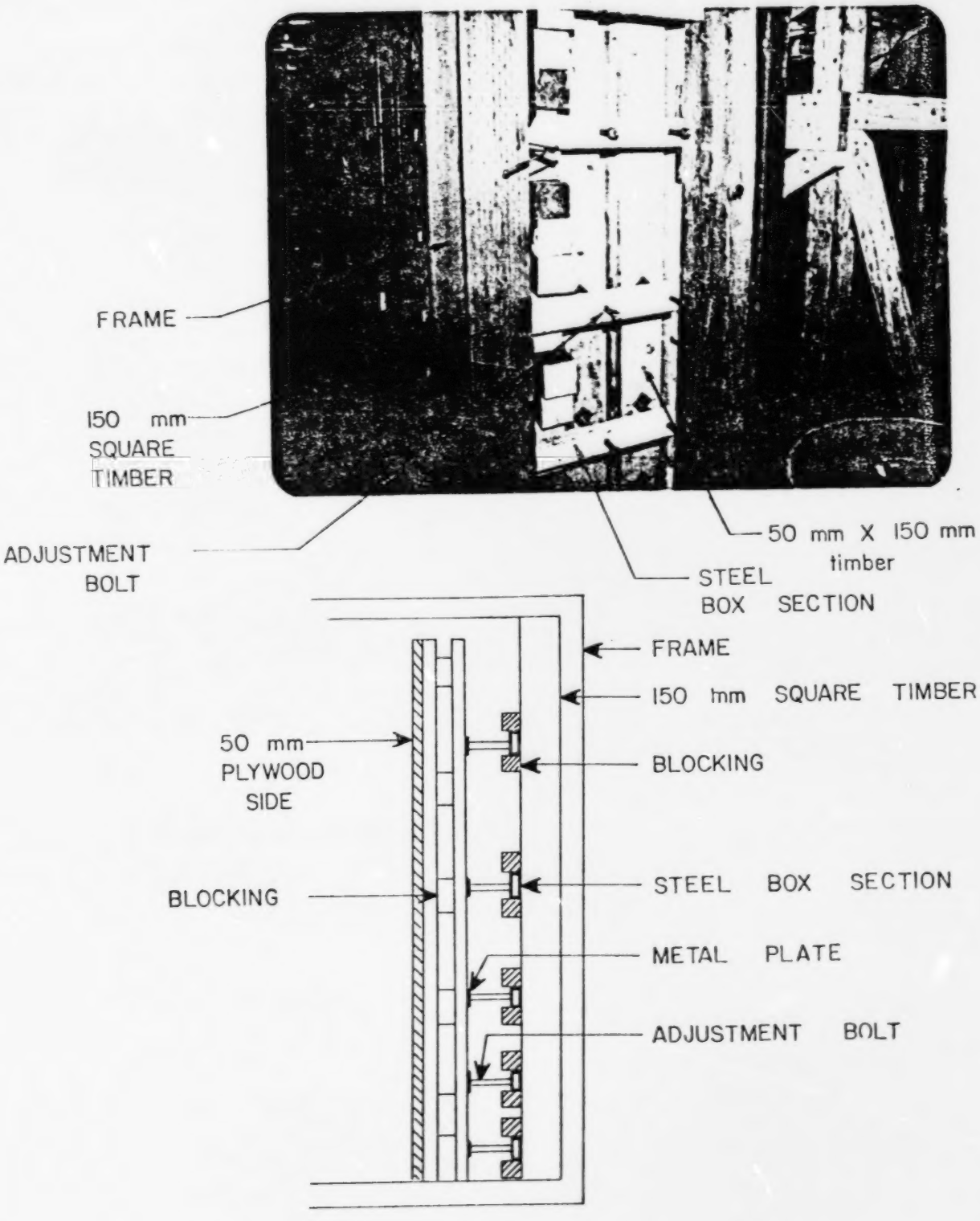


FIGURE 6 — SIDE WALL SUPPORT

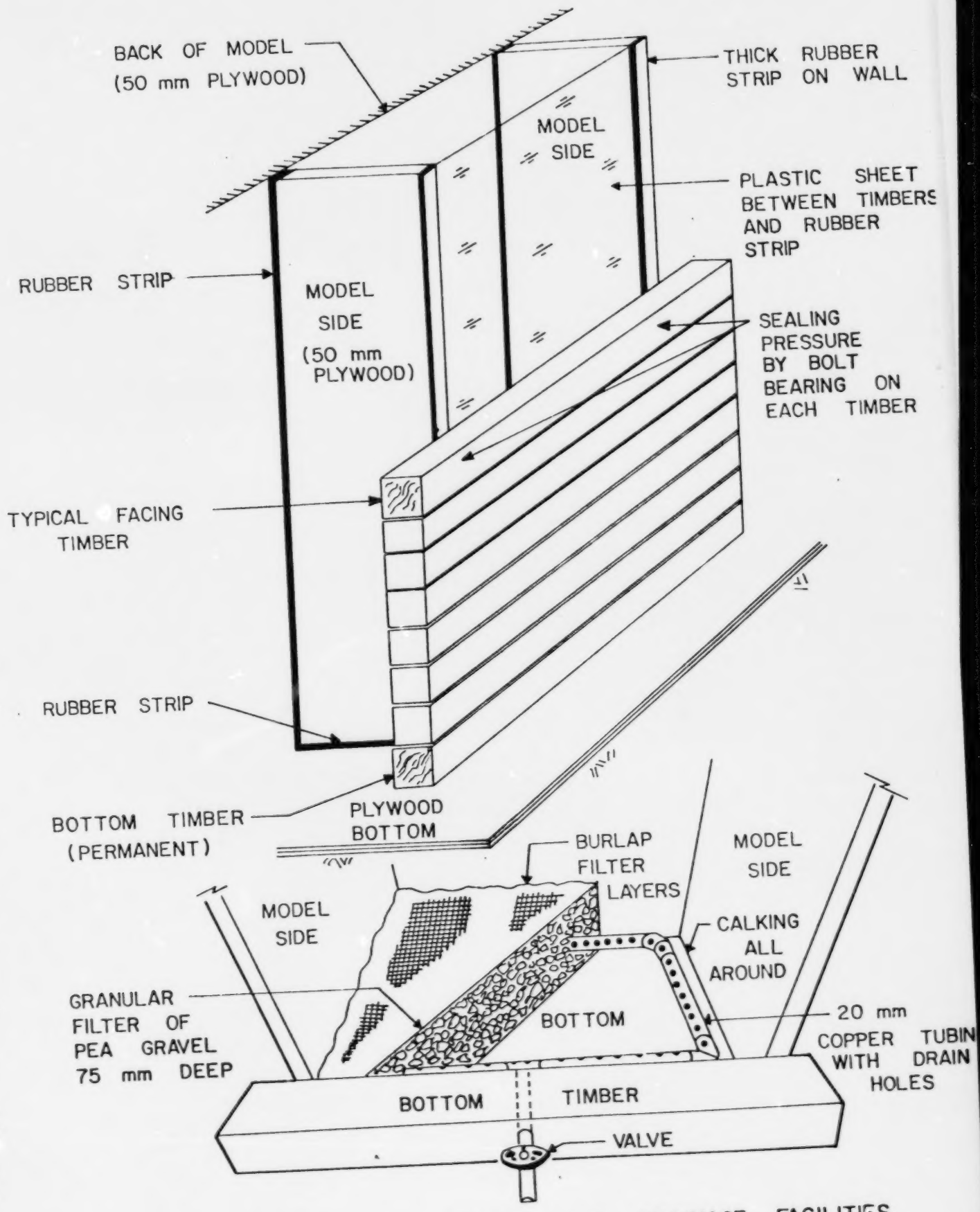


FIGURE 7 – SEALING AND DRAINAGE FACILITIES

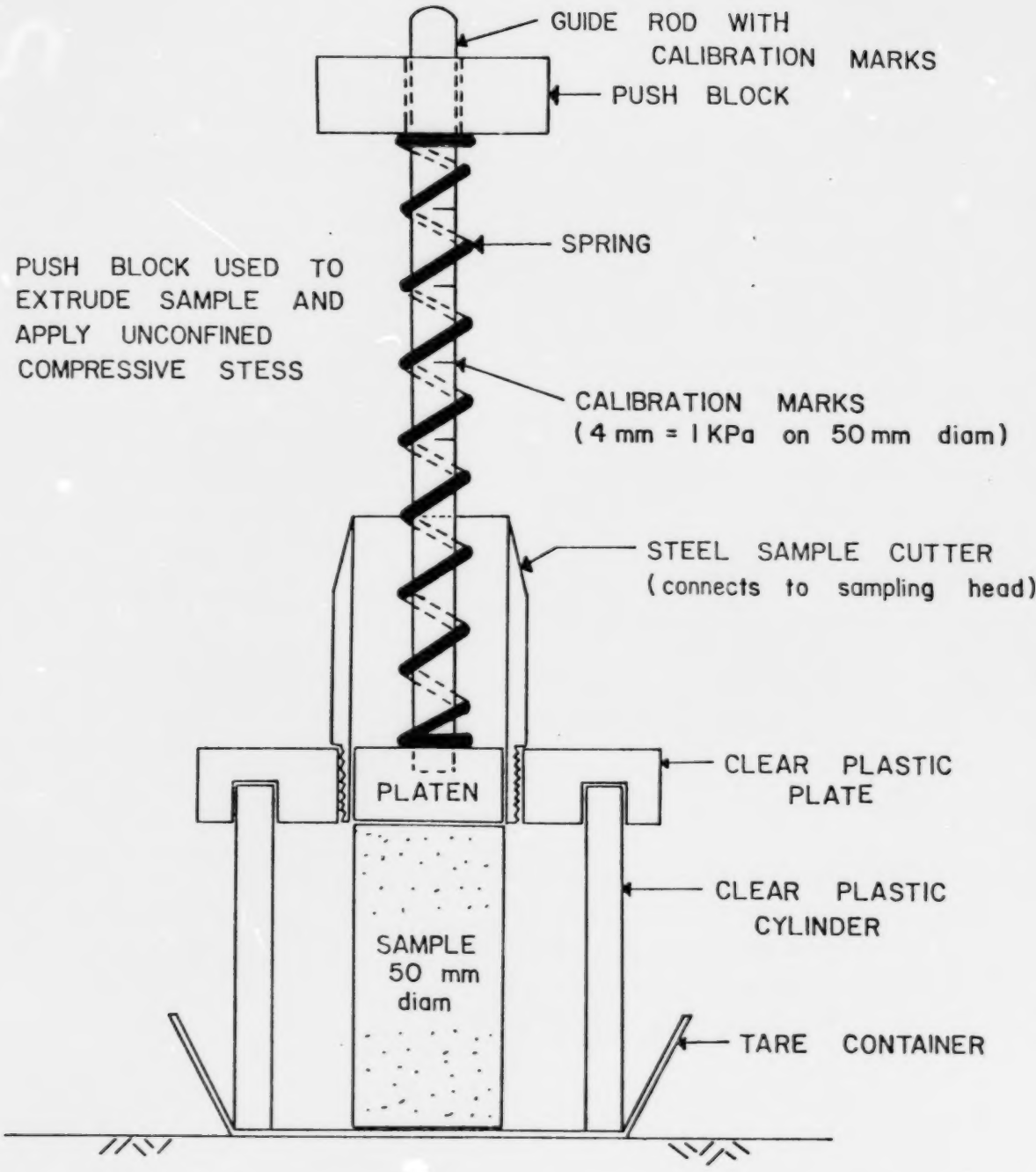

GUIDE ROD WITH CALIBRATION MARKS
PUSH BLOCK
SPRING
PUSH BLOCK USED TO EXTRUDE SAMPLE AND APPLY UNCONFINED COMPRESSIVE STESS
CALIBRATION MARKS (4 mm = 1 KPa on 50 mm diam)
STEEL SAMPLE CUTTER (connects to sampling head)
PLATEN
CLEAR PLASTIC PLATE
CLEAR PLASTIC CYLINDER
SAMPLE 50 mm diam
TARE CONTAINER

FIGURE 8 — UNCONFINED TEST APPARATUS

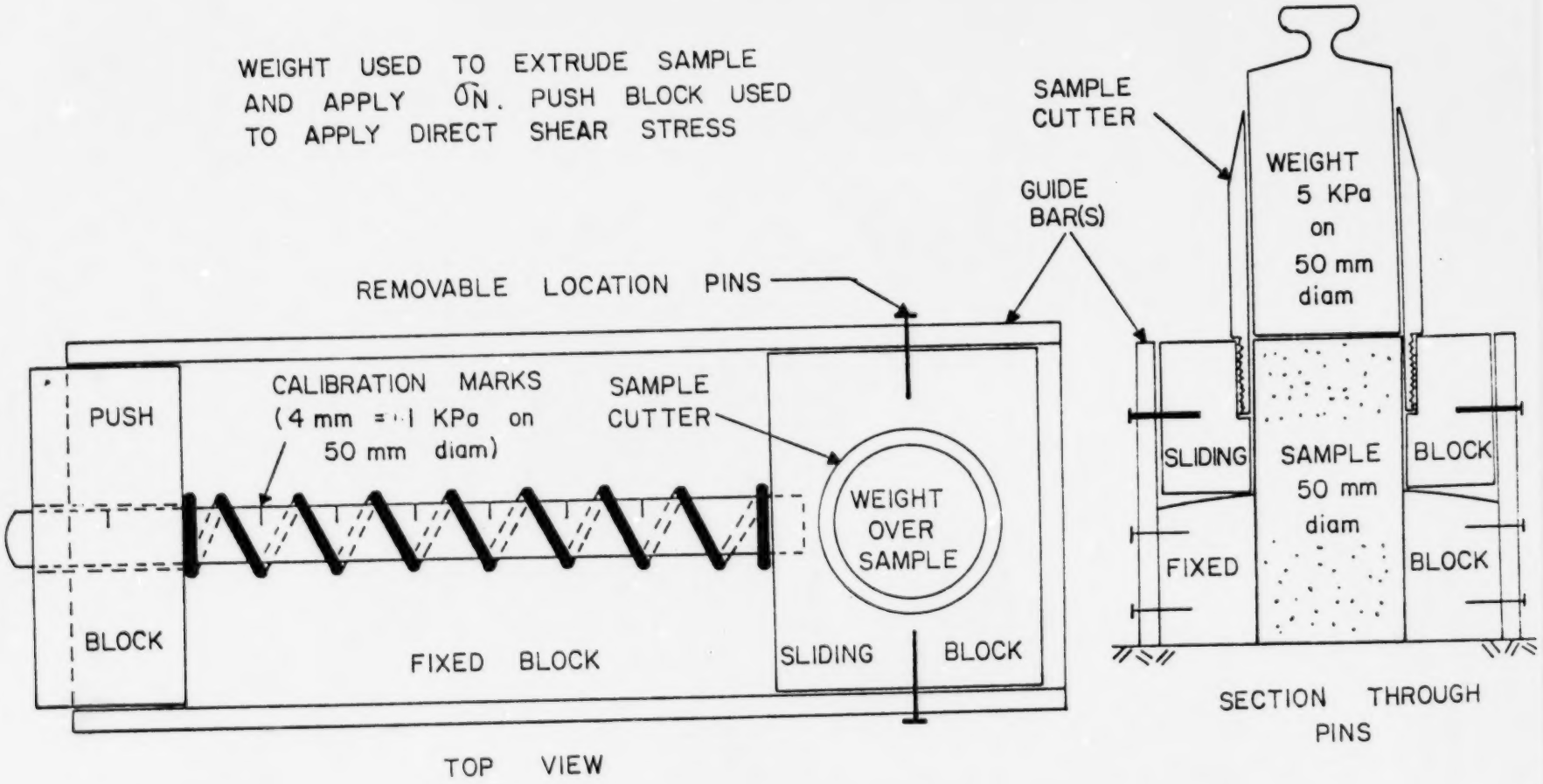


FIGURE 9 DIRECT SHEAR APPARATUS

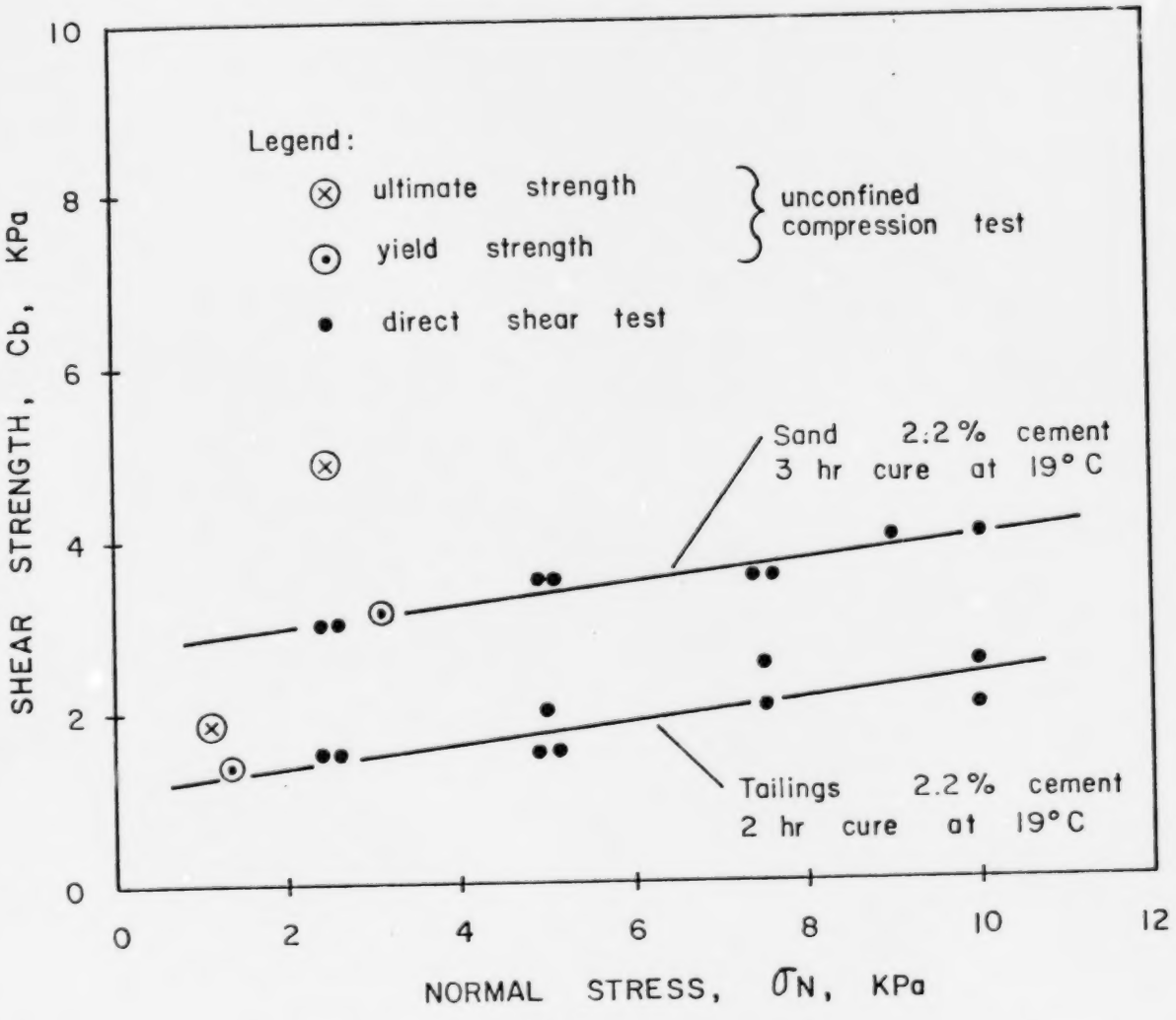
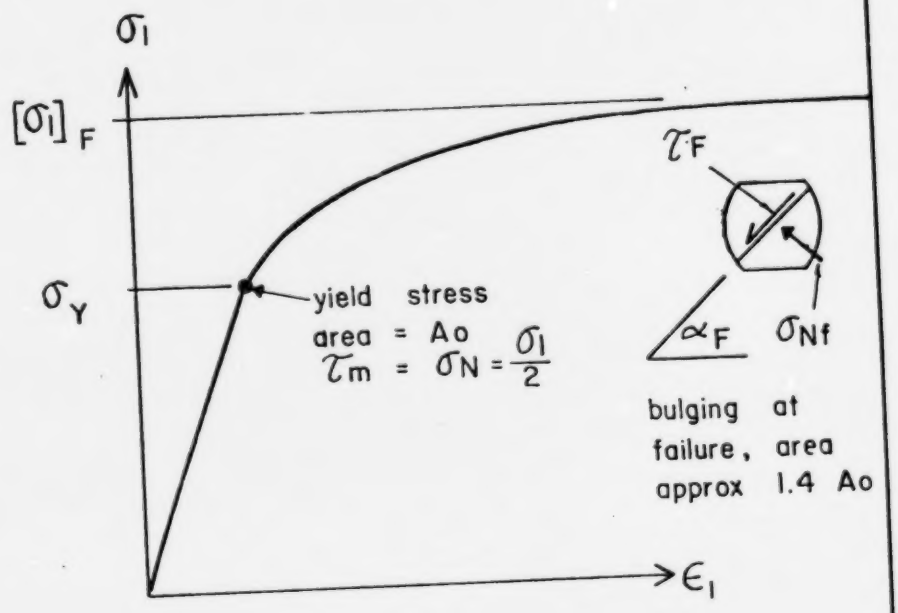


FIGURE 10 STRENGTH INCREASE WITH NORMAL STRESS



MATERIAL	σ_y KPa	$[\sigma_1]_F$ KPa	τ_f KPa	σ_{Nf} KPa
Tailings	2.8	3.6	1.7	1.0
plus 2.2 %	2.6	3.6	1.6	0.9
cement	3.2	5.2	2.1	1.3
2 hr cure	2.6	4.4	1.9	1.1
at 19° C	2.8	4.2	1.8	1.1
Sands	6.2	10.8	4.8	2.7
plus 2.2 %	6.4	10.8	5.0	2.8
cement	6.4	13.5	5.4	2.4
3 hr cure	6.6	8.8	4.0	2.2
at 19° C	6.4	11.0	4.8	2.5

FIGURE 11 - UNCONFINED TEST ANALYSIS

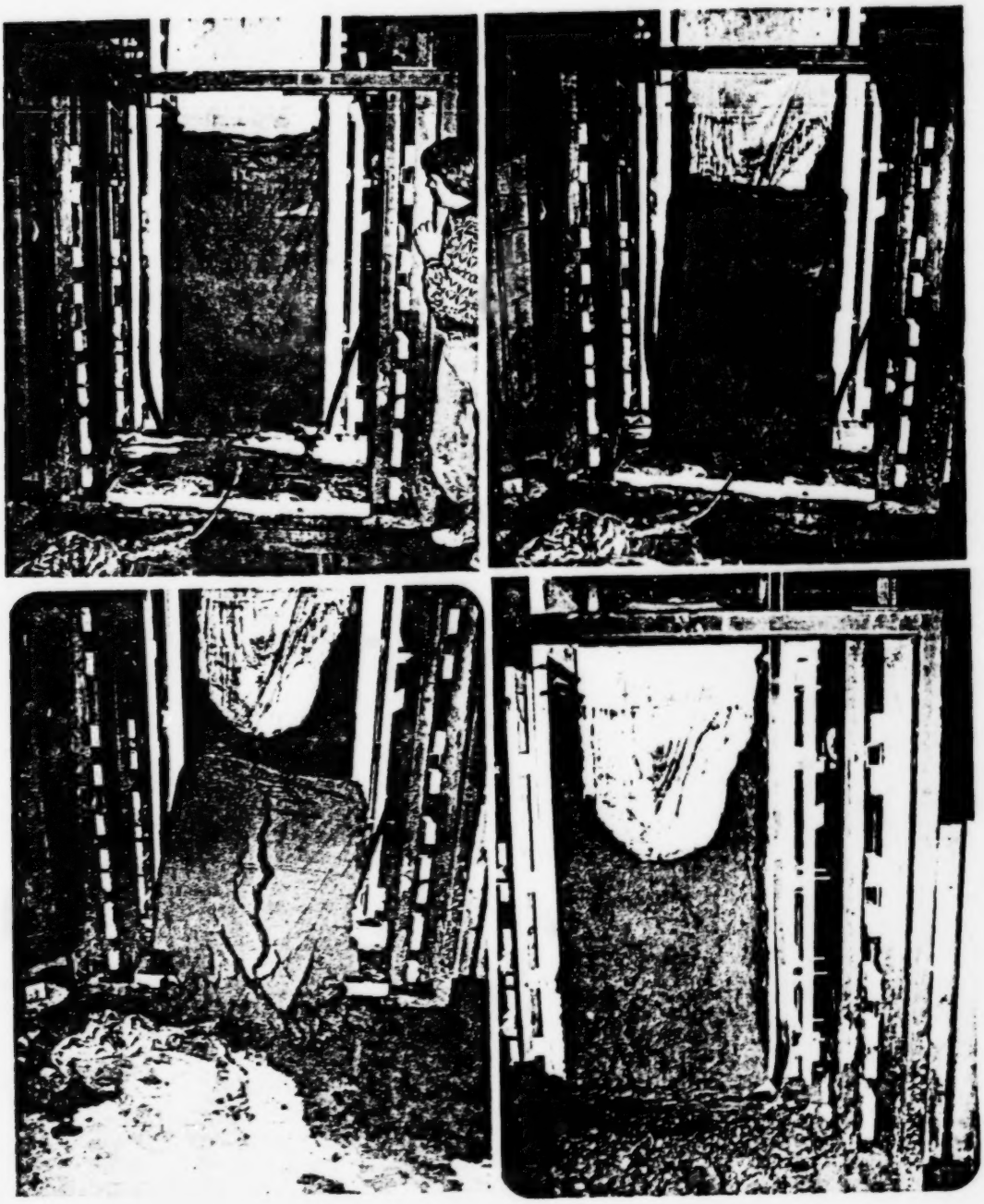


FIGURE 12 — PHOTO SEQUENCE, TEST T 11

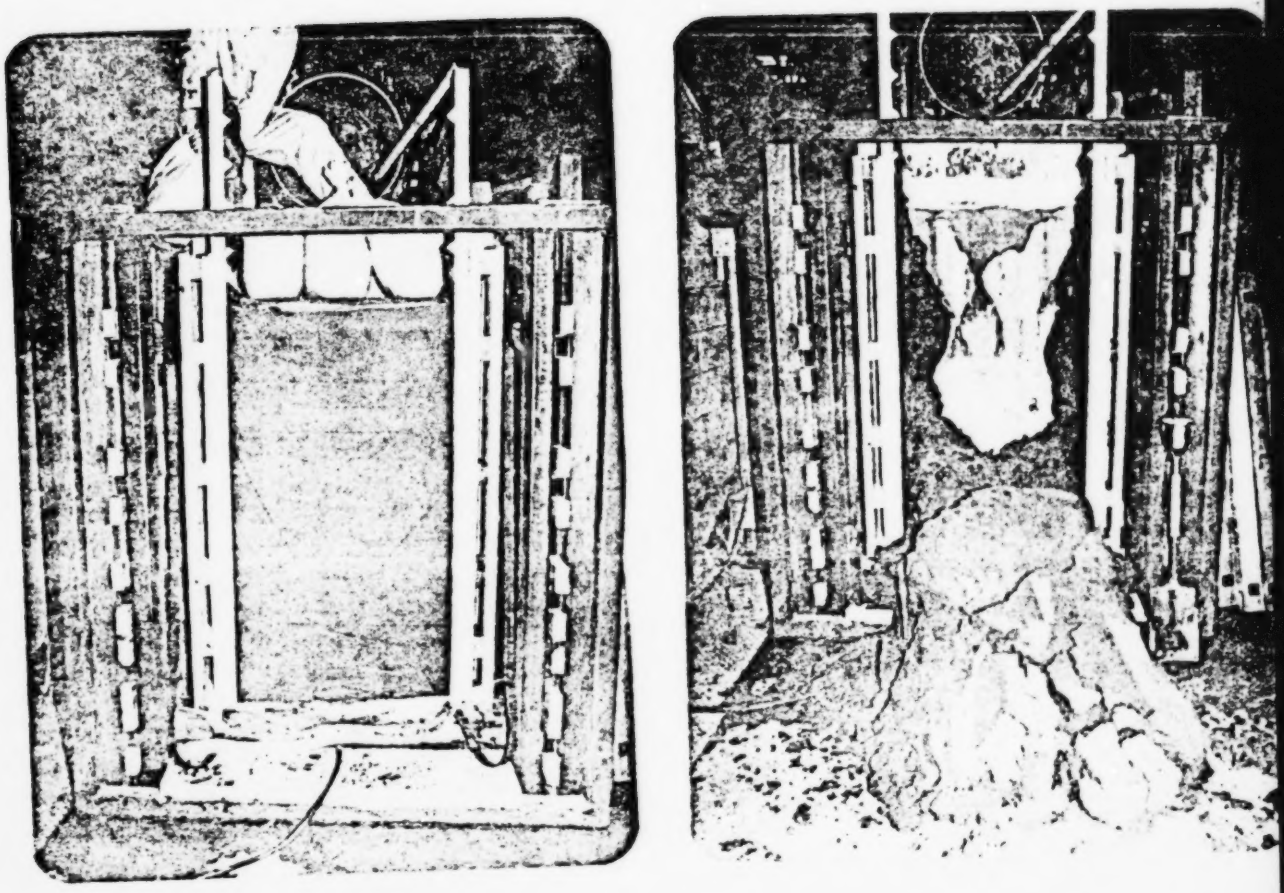


FIGURE 13 – FAILURE OF TEST T6 UNDER SURCHARGE

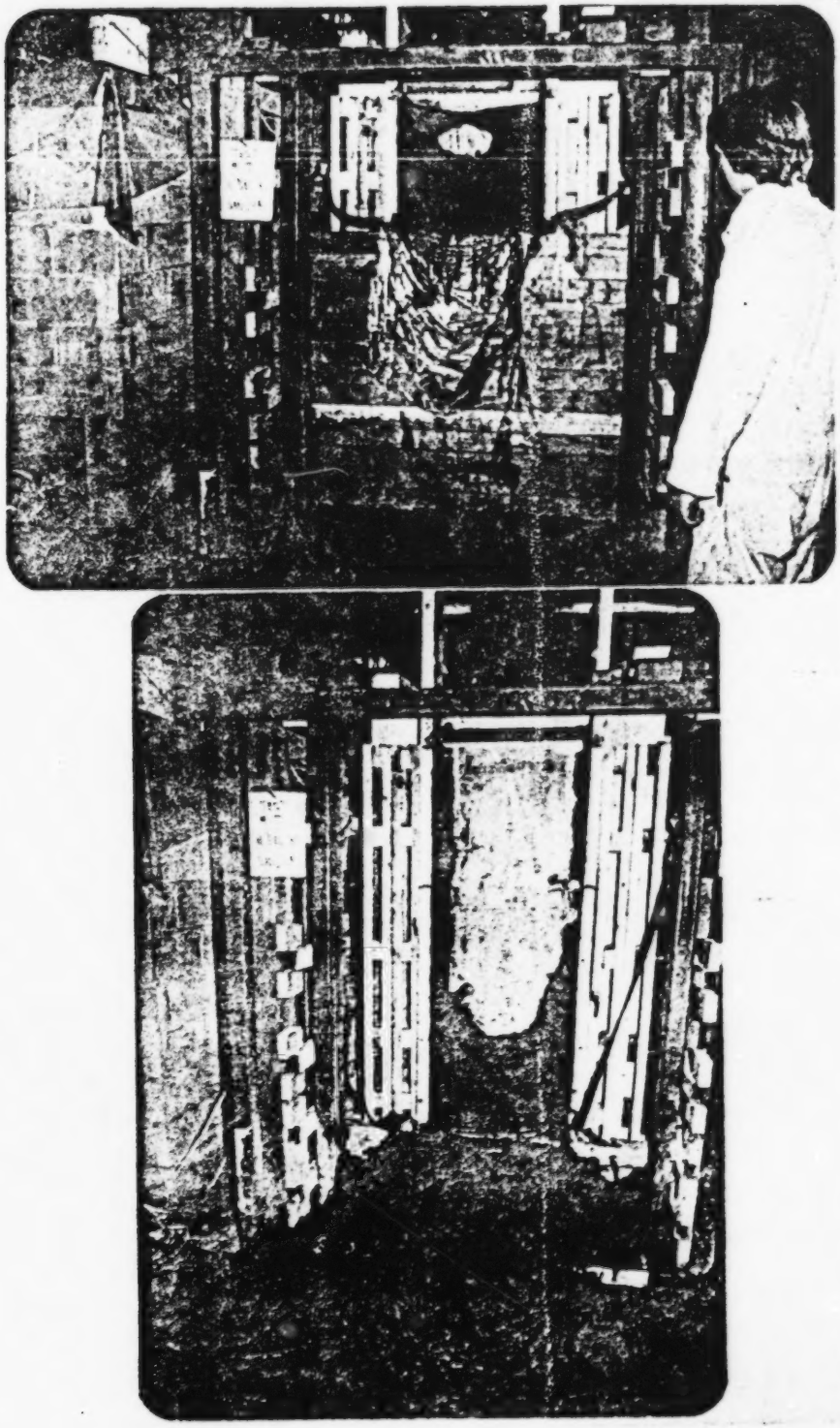


FIGURE 14 - DOUBLE FAILURE IN TEST S 17

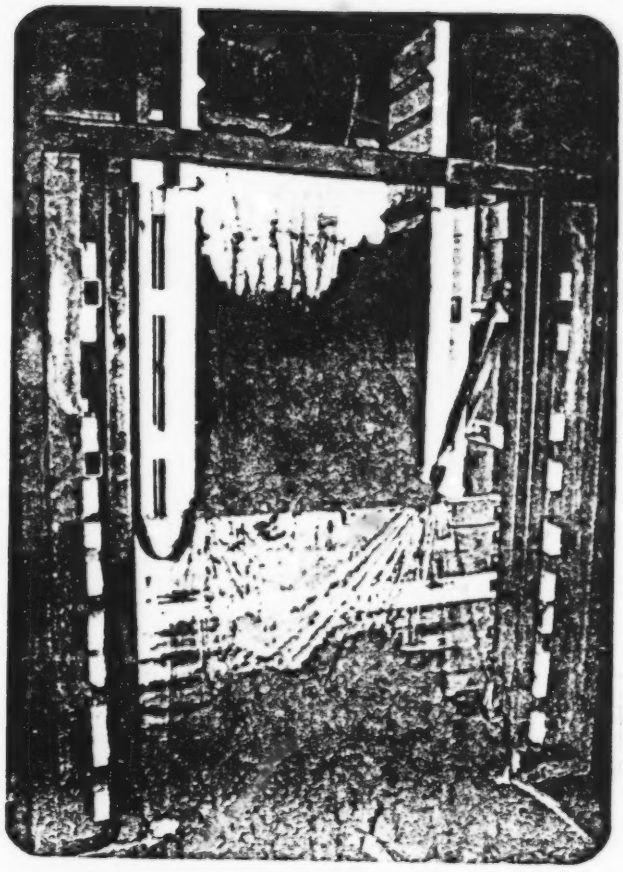


FIGURE 15 – FAILURE OF TEST T 10
AFTER CLOSURE STRAIN OF 1 %

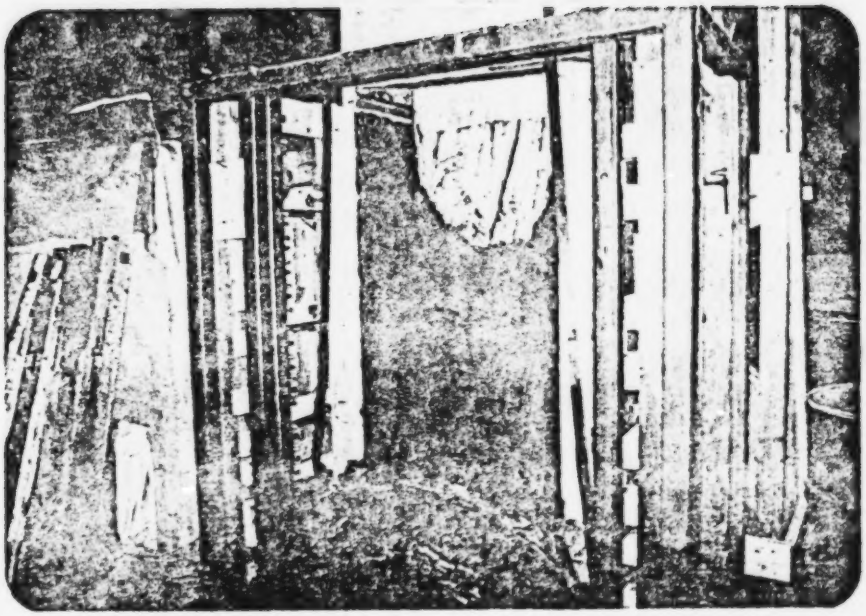


FIGURE 16 – FAILURE OF TEST T 9 : NO CLOSURE STRAIN, EXPOSED FROM TOP

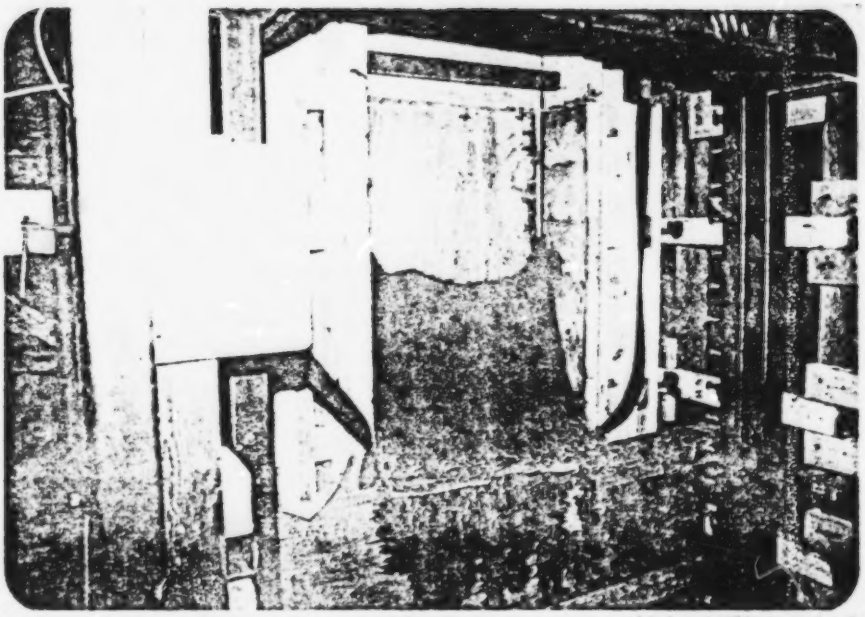
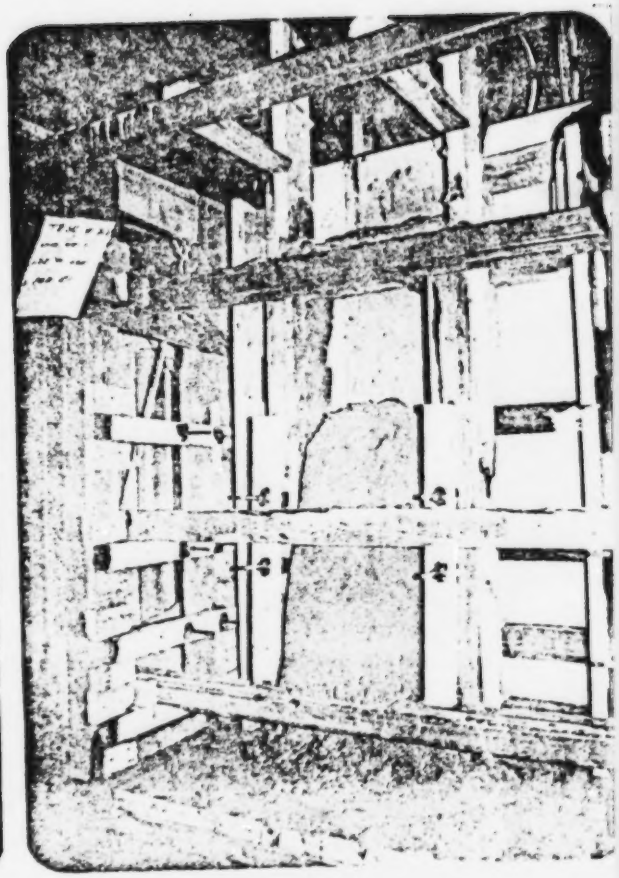


FIGURE 17 – FAILURE OF TEST T 18
WITH NO SIDE WALL ROUGHNESS

REMOVAL OF TIMBERS FROM SIDE TO SIDE



FAILURE OF FILL AFTER REMOVAL OF ALL BUT THE TWO OUTER EDGE TIMBERS

FIGURE 18 – FAILURE OF MODEL FILL EXPOSED FROM ONE SIDE (TEST S 28)

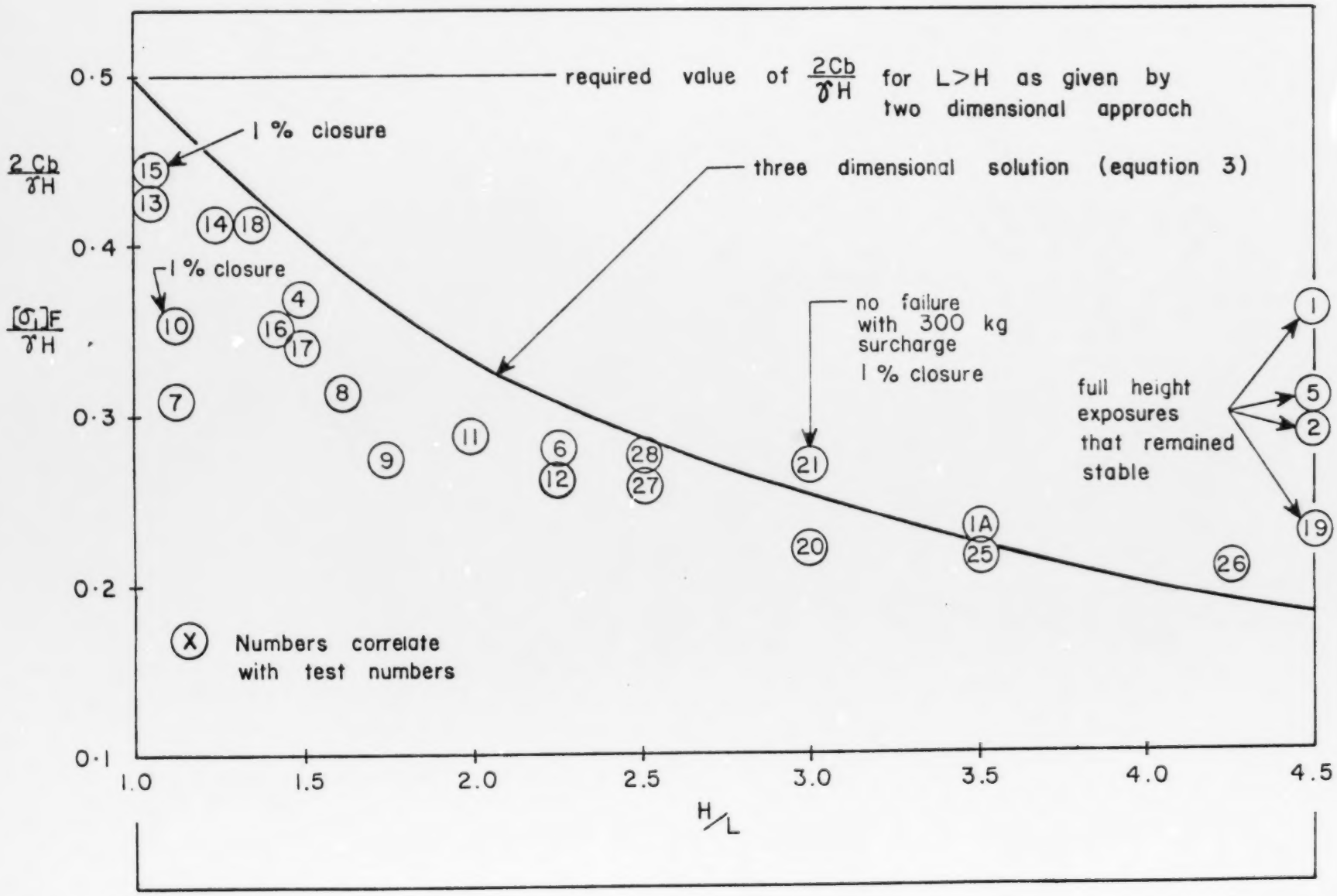


FIGURE 19 - CORRELATION OF TEST DATA WITH STABILITY ANALYSIS

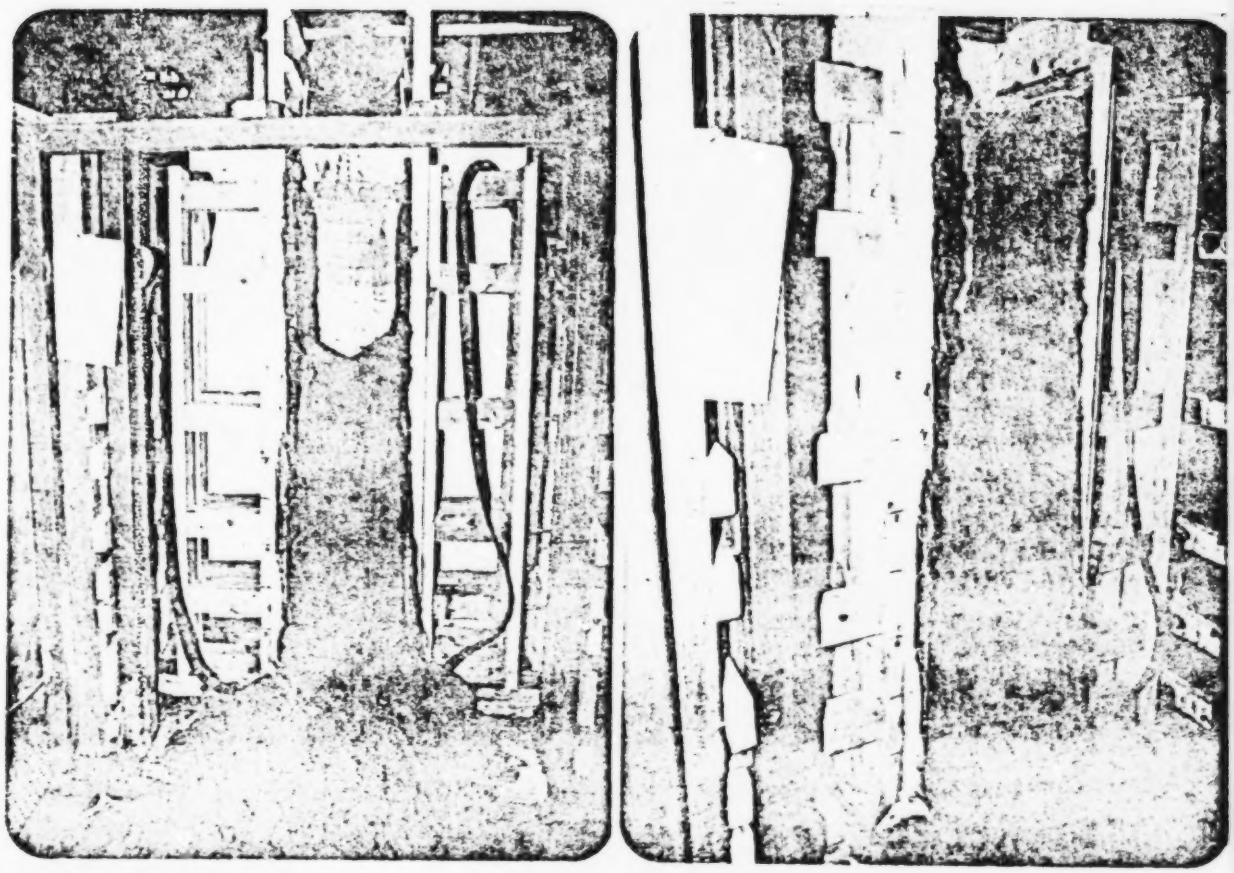


FIGURE 20 – FAILURE WITH HIGH VALUE OF H/L (TEST T 26)

PART B - TEST DETAILS FROM TESTS 17 THROUGH 28

Our phase 1 report included test data on all models up to and including test No. 16. The raw data from tests 17 through 28, carried out as specified in fulfillment of our contract, are included as Appendix A. Several problems were mentioned in the phase 1 report and these are discussed in detail in section 3.3 of Part A of this report. We would, in particular, draw attention to the laboratory test data reported on Figures 10 and 11 of Part A which were carried out in January in order to solve the problem of lack of correlation between direct shear tests and unconfined compression tests (17 direct shear tests and 8 unconfined tests are included in this data). These tests constitute the final phase of laboratory testing required to analyze the model test results. Part A also includes the theoretical concepts and test data analyses as required in completion of the contract.

Our only technical disappointment in the entire program is the fact that tests 22, 23 and 24 were not successful. The failures noted on the data sheets are due to internal erosion (sapping) of the coarse sand as can be clearly distinguished from the photo prints attached as Figure B. This problem and the solution is noted in section 3.3 of Part A but it was not a serious problem in any of the other tests. It is believed that the problem was caused in these three tests as a result of failure of the base drainage facilities and the problem was solved when the formwork dimensions were altered to proceed with test No. 25. Models 22, 23 and 24 were stable in the absence of erosional failures and the data would plot on Figure 19 as three stable points lying above test point 21 (from test No. 21). Since

the model curing time was down to the practical limit of two hours on these tests, there was no point in trying to repeat the 16" x 16" test size with the coarse sand material. More tests could have been carried out with the tailings material to obtain additional test points for $^{H}/_{L} > 2.5$ on Figure 19 but, unfortunately, we ran out of the tailings material and a trip to the borrow site convinced us that it would be impractical to try and obtain material at that time of year (January). We are, overall, quite pleased with the results and think that there is sufficient data on Figure 19 to convince most sceptics. If such testing is contemplated in the future, however, it should be carried out at over twice the scale (models 4 or 5 m. high) in order that longer curing times (approx. 24 hours) can be used. This would eliminate a lot of the minor problems (particularly the "guesswork" in deciding exactly when to expose the cured fill).

We hope that you share our pleasure in seeing this project through to a successful conclusion and we look forward to your comments and approval of this final report.

Respectfully submitted,

per

Robert J. Mitchell for
John D. Smith Engineering
Associates Limited.

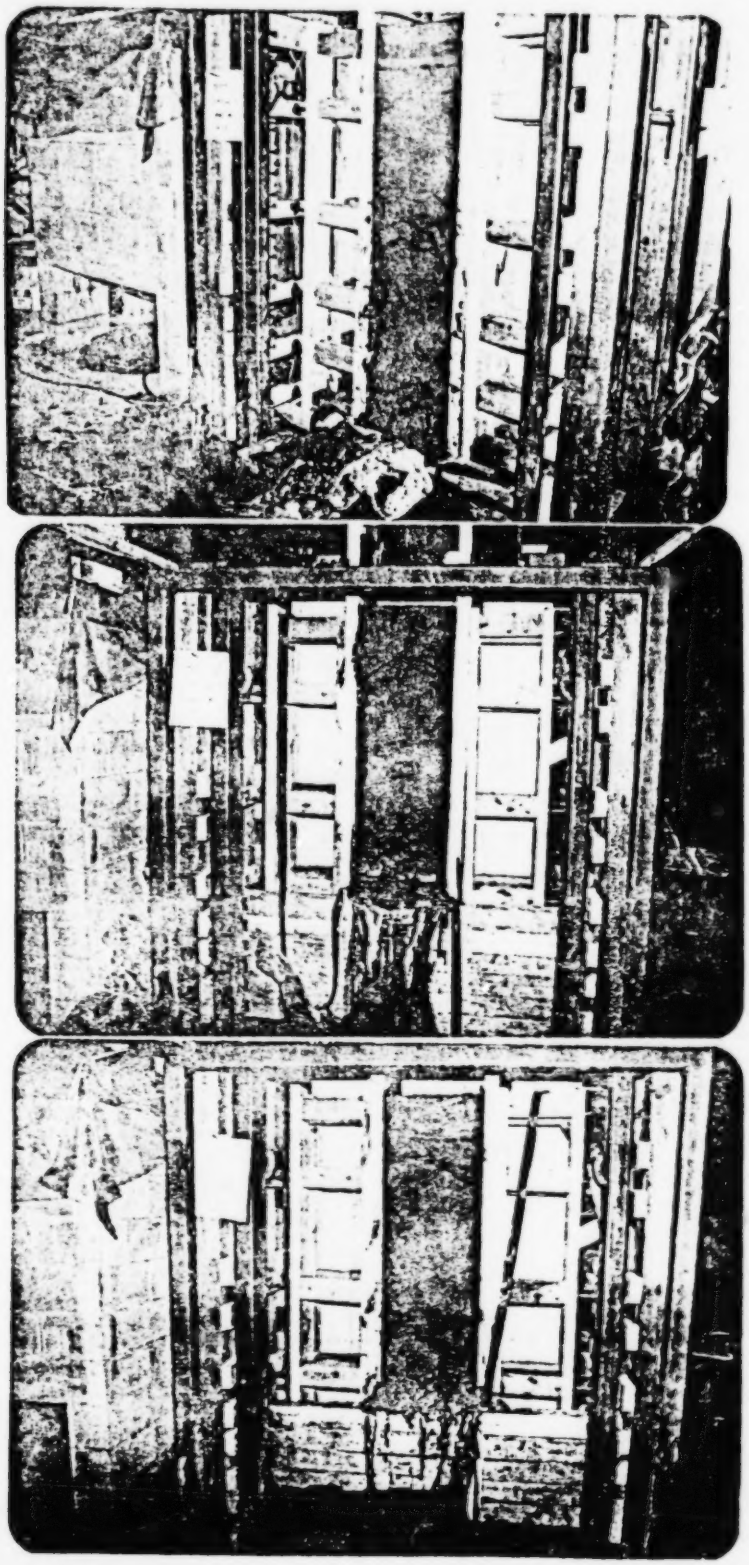


FIGURE B1 – SAPPING FAILURES